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Comparing rapid load pile testing for driven and CFA piles in London Clay

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Keywords: rapid load testing, driven piles, CFA piles, rate effects, clay

ABSTRACT: The current analysis of rapid load tests (RLT) such as Statnamic is normally based upon empirical correlations with static pile tests in similar soils. In certain soil types, such as clays the number of case studies used to develop analysis and allow selection of appropriate rate effect correction are limited. Due to these limitations selection of correction factors does not distinguish between pile type or pile installation technique. In clay soils it is well known that driven piles may have significantly enhanced capacity over cast insitu piles of similar cross-section. To test the effect of pile installation technique on RLT analysis RLT testing and static testing were undertaken on precast driven concrete piles and cast insitu CFA piles installed in high plasticity London Clay. Results show that the installation technique does not appear to affect the magnitude of the rate effects, provided modifications are made to the analysis to account for the previously reported differences in static capacity between different installation techniques. Based upon the findings it is suggested that RLT analysis should distinguish between pile type and installation techniques and for existing analysis techniques further case study based rate correction parameter are required, especially in clay soils.

1 INTRODUCTION

The analysis of rapid load pile testing (RLT) such as Statnamic is currently heavily dependent on the use of empirically derived damping or rate effect parameters to correct for the viscous effects in soil at elevated strain rates. Recent developments to RLT analysis include the selection of damping and correction parameters based upon soil type (Paikowsky, 2004, Middendorp et al. 2008) and measureable properties such as Atterberg limits in clays (Powell and Brown 2006).

Currently the rate effect parameters are derived from direct comparison of the RLT load-settlement behaviour with that of a static pile test on the same pile or an identical pile installed in close proximity. Alternatively the parameters may have their origin in high strain rate laboratory element testing (for example Schmucker 2005). Unfortunately in the former case there is a lack of high quality case study data upon which to confidently specify rate effect parameters especially in fine grained soils such as clays or silts. This has led to reluctance to specify correction parameters in clays (McVay et al 2003). This may result in a lack of end-user confidence in test results in fine grained soils and ultimately limits

further analysis development. Determining rate effect parameters from laboratory element testing is appealing from the point of view of consistency and control but historically testing has been undertaken at strain rates that are much lower than those experienced in RLT (Leinenkugel, 1976, Sheahan et al. 1996, Katti et al. 2003).

Although the effect of soil type on RLT analysis appears to have been recognised (Paikowsky, 2004, Powell and Brown 2006, Middendorp et al. 2008) the effects of pile type and installation technique has seen limited investigation. For instance in clay soils a driven pile (displacement) is likely to have higher static ultimate capacity than a pile of similar cross section and length installed by boring techniques and cast insitu (non-displacement). The effect on pile shaft capacity of the method of installation is well documented with bored piles displaying approximately 70% of a driven pile's shaft capacity (Fleming et al. 2009). This is also reflected in the higher adhesions factors used in total stress design for driven piles (Weltman and Healy, 1978). It is not currently clear if an associated increase in pile resistance would be measured during an RLT test and therefore allow the use of the same correction parameters for both displacement and non-displacement piles.

Due to the tendency for increased static capacity of displacement piles in clay it is therefore necessary to investigate this effect on both RLT analysis and parameter selection. For instance the technique proposed by Schmuker (Krieg and Goldscheider 1998, Schmuker 2005, Middendorp et al 2008) has its origins in low strain rate laboratory element testing which cannot easily replicate pile-soil interface behaviour, complicated variations in insitu effective stress or the effects of the high soil strain levels encountered during pile driving. The analysis method proposed by Powell and Brown (2006) and Brown and Hyde (2008) derives the majority of its soil dependant rate parameters from both back analysis of RLT field studies on non-displacement cast insitu piles and high strain rate (push-in) probing tests (Brown 2008).

To investigate the effect of pile installation technique and increase the available case study information for RLT in fine grained soils a series of driven precast piles were installed at a research site underlain by London Clay. The results of RLT and static testing of these piles was compared with the results from testing cast insitu continuous flight auger (CFA) piles installed at the same site.

This paper is a shortened version of that submitted to the accompanying special edition of JGS Soils and Foundations Journal (Brown and Powell 2012^a).

2 FIELD STUDY SITE

The study site is located at Lodge Hill Camp, Chattenden, Kent in the UK and is underlain by London Clay to a depth in excess of 35 m. The upper 4 m is typically weathered/desiccated brown

London clay (OCR 50 to 24) which overlays unweathered blue clay of very high plasticity. The undrained shear strength in the upper 10 m gradually increases with and average shear strength of 100 kPa (average OCR 18). The plasticity index, $PI = 60\%$ in upper 10 m, rising to 63% for 10-15 m. The average moisture content in the upper 15 m was 29% and the bulk density, $\gamma = 19.4 \text{ kN/m}^3$. The water table was at approximately 1 m depth. The Soil strength and characterisation data are shown summarised in Fig. 1.

3 PILES AND TESTING REGIME

Pile testing was undertaken on driven precast piles and CFA piles installed at the site. The precast driven piles were 11.0 m long, driven to a depth of 10 mBGL and had a square cross section of 275 mm×275 mm. The cast in-situ 450 mm diameter CFA piles were installed to a depth of 10.8 mBGL with an effective length of 9.667 m due to extension casing installation. The CFA piles were extended above ground at the time of casting by adding an 11 mm thick steel casing of 500 mm diameter filled with concrete. The design or characteristic static load capacity ($F_{u, \text{design}}$) of both types of pile was approximately 1000 kN.

In total four precast driven piles and seven CFA piles were tested in the study. For each pile type “identical” piles were installed and reserved for testing by a specific technique e.g. one pile would have exclusively RLT tests undertaken on it and compared with static tests on an adjacent pile rather than both types of test on one pile.

3.1 Static pile testing

Static pile tests were performed using a hydraulic jack reacting against a frame restrained by anchor piles with loads measured directly using a calibrated load cell. The test procedure complied with the ICE Specification for Piling and Embedded Retaining Walls (SPERW) (ICE 2007)

Two driven precast piles were tested to prove ultimate loads, one with a maintained load procedure (ML) (TP1) followed by a constant rate of penetration stage (CRP). The second pile (TP2) was tested just using CRP procedures (Fig. 2). The test procedure employed for the ML test on the driven pile TP1 was to increase the loads in 125 kN increments with unload/reload cycles at 500, 750 and 1000 kN ($0.5, 0.75, 1.0 \times F_{u, \text{design}}$). The CRP tests for piles TP1 & 2 were undertaken at an average constant rate of 0.01 mm/s until a peak load had been reached. At this point the rate of loading was increased to the safe maximum of the system, resulting in typical average settlement rates 0.103 mm/s (referred to as CRP(H) and labelled as

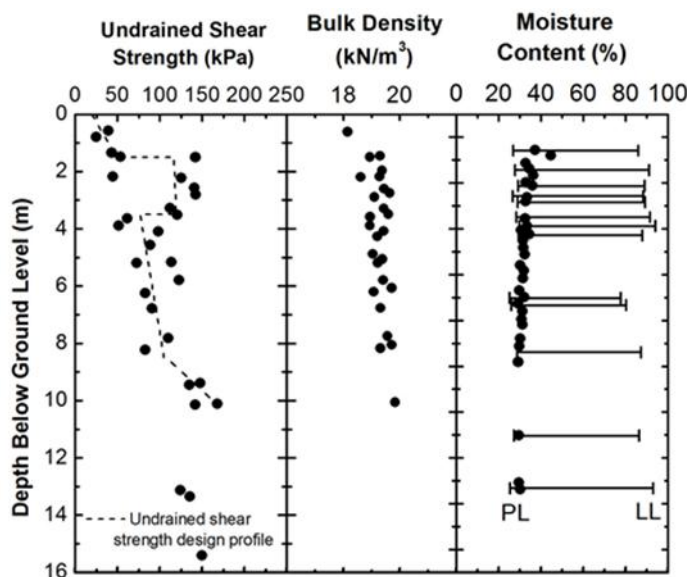


Fig. 1. Typical soil characteristics for the Chattenden site (Brown and Powell 2012^b)

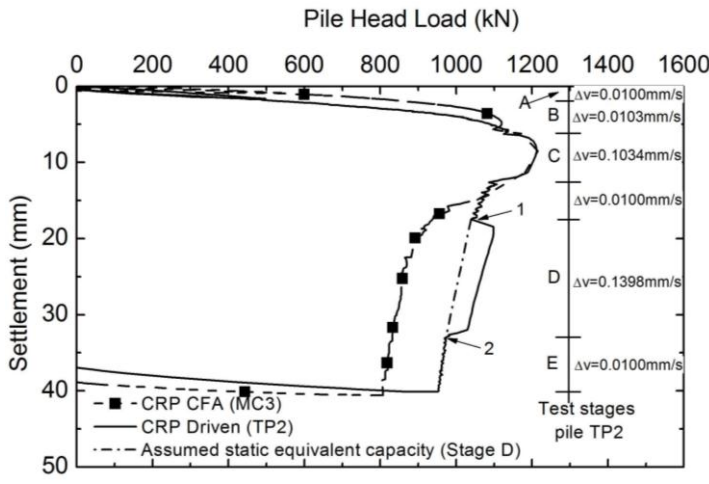


Fig. 2. Comparison of static CRP testing for a driven precast pile (TP2) and a CFA cast insitu pile (MC3). Stages A to E refer to variation in pile penetration rate for pile TP2 only (Table 1), (Brown and Powell 2012^a)

C and D in Fig. 2), for a short period to assess the effect of the rate of loading on the ultimate capacity. A similar approach to static testing was adopted for the CFA piles.

3.2 Rapid load pile testing

Rapid load testing (RLT) consisted of Statnamic testing undertaken using a 4 MN rig with a hydraulic catch mechanism. For both types of pile several cycles of RLT loading were applied in quick succession on the same pile with each cycle increasing in magnitude. The selection of load cycle magnitude generally followed the pattern of 0.75, 1.0, 1.5, 1.7, 2.5 times the static design load for the driven precast piles (Fig. 3). The selection of load cycle magnitude was less systematic for the CFA piles and was varied as each pile was tested to produce significant settlement (Fig. 4). For these piles the RLT load cycles varied between 0.87 to 4 times the static design capacity.

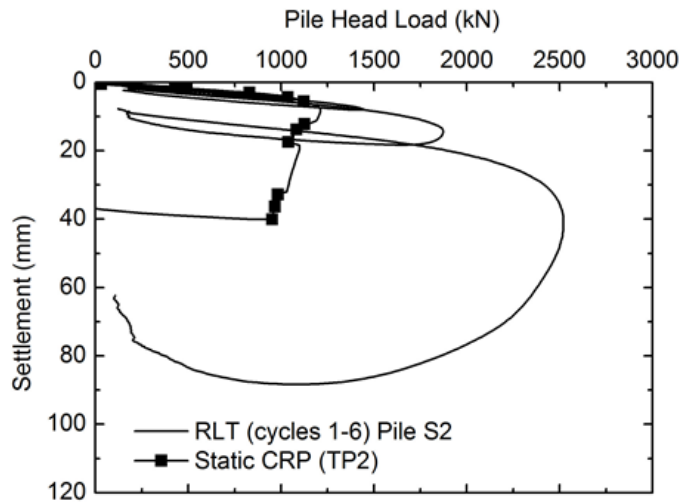


Fig. 3. Comparison of RLT load cycles (S2) with CRP static testing (TP2) for the precast driven piles (Brown and Powell 2012^a)

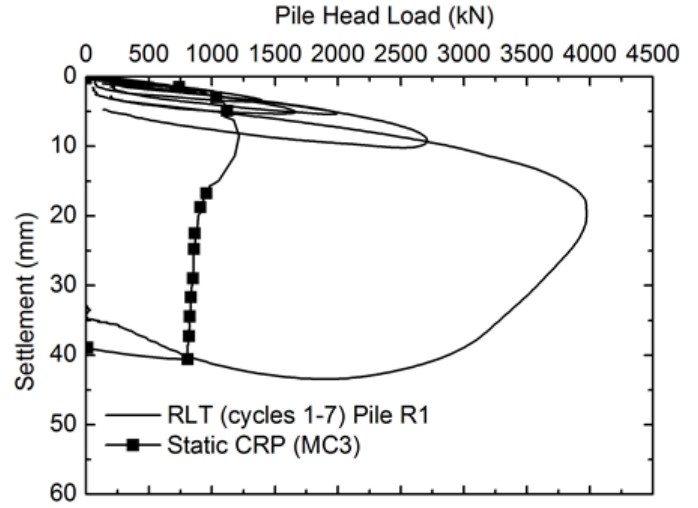


Fig. 4. Comparison of RLT load cycles (R1) with CRP static testing (MC3) for the cast insitu CFA piles (Brown and Powell 2012^a)

4 DISCUSSION OF RESULTS

4.1 Results of static testing

Typical results of the static CRP pile testing are compared for the CFA cast insitu pile MC3 and the driven pile TP2 in Fig. 2 with key results summarised in Table 1. For reference purposes certain key features or stages on the graphs are referred to using the letters A to E. Stage A refers to the first cycle of standard rate CRP testing to approximately half of the static design load. At this point the settlement of the CFA pile was approximately half that of the driven pile which is to be expected based upon the reduced cross section of the precast pile. Stage B indicates the initial peak bearing capacity reached for both piles at standard settlement rates which is of a very similar magnitude (Table 1) and highlights the enhancement of pile capacity due to the difference in installation techniques. Pile settlement is also reported at 495 kN in Table 1 which reflects working load settlements with 495 kN selected as a common load level encountered in Stage A of the two CRP tests reported. For example assuming a simple total stress analysis for shaft friction resistance (F_{shaft}) where

$$F_{shaft} = \alpha \times s_u \times A_{shaft} \quad (1)$$

$$F_{base} = N_q \times s_u \times A_{base} \quad (2)$$

where N_q is assumed to be 9 and A_{shaft} and A_{base} refer to the surface area of the pile shaft and the area of the pile base respectively.

Back analysis of the standard rate static load test data gives an adhesion factor $\alpha = 0.98$ (average unit skin friction = 95 kN/m²) for the driven pile and 0.73 (average unit skin friction = 69 kN/m²) for the CFA pile at peak capacity i.e. an adhesion factor

Table 1. Comparison of static tests on driven and CFA piles

Pile	Stage ¹	Test type	Max. applied load (kN)	Δ_h at 495 kN (mm)	Average penetration rate (mm/s)
MC3	A	CRP	540	0.74	0.0102
	B	CRP	1120	0.61	0.0096
	C	CRP(H)	1215	-	0.1676 ² (0.2146) ²
	E	CRP			0.0120
TP2	A	CRP	497	1.64	0.0100
	B	CRP	1138	1.34	0.0103
	C	CRP(H)	1212	-	0.1034 ² (0.1450) ²
	D	CRP(H)	1099	-	0.1398
	E	CRP			0.0100

¹ The stages correspond to the labels on Fig. 2

² Unable to maintain a constant penetration rate, peak shown in parenthesis

ratio of 0.75 (= 0.73/0.98) between the driven and cast insitu piles which is slightly lower than the ratio of 0.8 suggested by Fleming et al. (2009). The increased adhesion factor for driven piles is consistent with the findings of Weltman and Healy (1978) and Bond and Jardine (1991).

Stage C shows the effect of the increased settlement rate associated with the CRP(H) test on the two pile types. The high settlement rate peak strength is almost identical for the two pile types. As the peak capacity at the standard rate was very similar (Stage B) for the two piles (Table 1) this would appear to show that the enhancement of capacity with increased settlement rate is also similar suggesting that the magnitude of rate effect is unaffected by the installation technique (over the range of penetration rates investigated). If rate enhancement of the pile tip component is ignored (Brown 2004) this suggests an average increase in shear strength on the shaft from 95 kPa to 103 kPa. Fixing the undrained shear strength at the initial insitu values the adhesion factor increases to 1.05 ($\alpha = 0.98$ at standard rate) and 0.81 ($\alpha = 0.73$ at standard rate) for the driven and CFA piles respectively.

As the settlement rate varies slightly between the CRP(H) on the driven and CFA piles it is useful to introduce a relationship that allows the representation of the rate effect whilst normalising for the pile settlement rate or velocity. The approach shown in Eq. (3) was developed by Randolph (2003) to represent pile shaft capacity enhancement during pile driving:

$$\tau_{lim} = \tau_s \left[1 + m \left(\frac{\Delta v}{v_0} \right)^n \right] \quad (3)$$

where τ_{lim} is the limiting elevated rate shaft friction, τ_s the static shaft friction, m and n are viscous

parameters and Δv is the relative pile-soil velocity, normalised by v_0 (taken as 1 m/s). For clay soils n is normally set to 0.2. To compare the rate effect the viscous parameter m has been back calculated using Eq. (3) which normalises variation in settlement rates and static pile capacity. The resulting variation of m for the two pile types is shown in Fig. 5. The process used to back calculate m can be understood by considering Stage D shown in Fig. 2 for pile TP2. In this case τ_{lim} is the unit skin friction measured during stage D at the elevated rate of penetration (Δv). The magnitude of τ_s is determined by calculating the shaft resistance for the equivalent static rate (v_0) test during this phase. This is achieved by considering the static shaft resistance just before the rate is increased (point 1, Fig. 2) to that associated with τ_{lim} in Stage D and at the end of Stage D when the rate of penetration again returns to the standard rate (point 2). Between these two points an equivalent static pile resistance variation is assumed as shown in Fig. 2. In turn this is used to determine an assumed static pile resistance variation (τ_s) which is used in the back calculation of m .

At the low settlements associated with peak pile capacity (Stage C) the value of m is identical for both types of pile. Again as settlement increase (Stage D) the CRP(H) tests show similar initial values of m although they appear to reduce rapidly for the CFA pile. This appears to suggest that the viscous rate effects are initially the same for the two types of pile installation and the rate effect itself is not affected by pile type/installation technique. Although the behaviour is initially similar the viscous parameter reduces significantly after the initial peak with increasing settlement or strain for the CFA pile. This may purely be an artefact of the testing and/or analysis employed or it may reflect the fast shearing and level of strain the soil around the driven pile has experienced during installation.

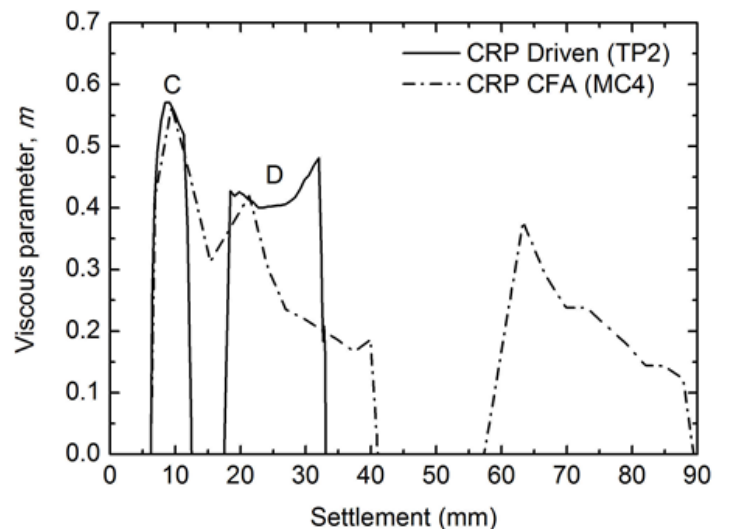


Fig. 5. Comparison of viscous rate parameters from elevated rate CRP(H) testing of the driven (TP2) and CFA piles (MC4), (Brown and Powell 2012^a)

On reducing the settlement rate to the standard rates associate with stage A and B (Fig. 2) both pile installation types show strain softening behaviour with this being greater for the CFA pile where the ultimate bearing capacity at the end of loading (Stage E) is 72% of the low settlement rate (CRP) peak capacity. The strain softening behaviour is not as marked for the driven pile with ultimate capacity being 84% of the peak load from CRP. This reduced degradation is likely to be due to the lower component of tip capacity and the preferred orientation of platy clay particles to form well defined shear planes for the driven pile.

4.2 Results of rapid load pile testing

Results of RLT loading on the CFA piles and driven piles are shown in Fig. 3 and 4. It is apparent that significantly larger loads need to be applied to the piles during RLT loading to achieve equivalent or greater settlements created during static loading and to fully mobilise the piles. For example in Fig. 3 for the driven piles the peak applied RLT load for cycle 6 which causes the largest settlement is 2521 kN (S2). This is 2.22 and 2.56 times the standard rate peak (Stage B) and ultimate (Stage E) capacities determined during the CRP static test. The maximum settlement rate of the pile during the RLT test was 2620 mm/s which compares to 0.01 mm/s during the CRP test. By comparison, to achieve significant settlement for the CFA piles (Fig. 4) a load of 3976 kN (R1) was applied which is 3.55 and 4.35 times the standard rate peak (Stage B) and ultimate (Stage E) capacities determined during the CRP test (MC2). At peak loads this would suggest that the apparent rate effects for the driven pile are approximately 72% of those for the CFA pile although it is difficult to make direct comparison as the maximum pile settlement rate (1293 mm/s) for the CFA pile was approximately half that during RLT of the driven pile (2620 mm/s).

4.3 Rapid load test analysis

Several methods have been developed to analyse RLT tests which aim to derive the static equivalent load-settlement behaviour through removal of both inertial and soil dependant rate effects. These are commonly referred to as the unloading point method (UPM, Middendorp et al. 1992, Middendorp 2000) and the Schmucker method (Schmucker 2005, Middendorp et al. 2008). Brown and Hyde (2008) proposed a non-linear velocity dependant technique (referred to simply as the Brown method) based upon Eq. (3) of the form:

$$F_u = \frac{F_{STN} - Ma}{1 + \left(\frac{F_{STN}}{F_{STNpeak}} \right) m \left(\frac{\Delta v}{v_0} \right)^n - \left(\frac{F_{STN}}{F_{STNpeak}} \right) m \left(\frac{v_{min}}{v_0} \right)^n} \quad (4)$$

Where F_u is the derived static pile resistance, F_{STN} is the measured Statnamic load where the subscript peak denotes the peak load measured during the RLT test, Ma is the pile inertia, Δv is the pile's velocity relative to the soil and v_{min} is the velocity of the static CRP pile test used to define the soil specific rate parameters m and n . The parameter n is normally set to a value of 0.2 for clay soils (Randolph and Deeks 1992). It has been proposed that the value of m may be linked to soil plasticity (Brown and Powell 2006) by the relationship:

$$m = 0.03PI(\%) + 0.5 \quad (5)$$

Schmucker (2005) proposed a soil specific analysis technique which relies on the selection of a soil viscosity index parameter I_{va} .

$$F_u = (F_{STN} - Ma) \cdot (0.02mm/min/\Delta v)^{I_{va}} \quad (6)$$

Where the viscosity parameter is related to a simple description of the soil as shown in Table 2.

The unloading point method is described in detail by Middendorp et al. (1992). Unfortunately when this technique is applied to piles installed in fine grained soils there is a tendency for the ultimate pile capacity to be significantly over predicted. In order to correct for this effect a series of soil dependant average correction factors were developed by which the derived static load multiplied to obtain a corrected UPM analysis (Paikowsky, 2004). The proposed UPM correction factor (μ) for clay of 0.65 is reported to be based upon a very limited number of cases (McVay et al. 2003). More recently it has been proposed that a much greater average correction factor in clay is required resulting in a μ value of 0.47 (Weaver and Rollins 2010).

The results of analysis using the procedures are shown in Fig. 6 for the CFA pile where the viscous rate parameter m in the Brown method has been set at 2.3 based upon the soil plasticity. The results of UPM analysis are shown corrected by both 0.47 and 0.65. In applying the Schmucker method a value for the viscosity index of 0.06 (Table 2) was used assuming that reference to bentonite in the table suggest a clay of very high plasticity.

Table 2. Soil viscosity parameters (Middendorp et al. 2008)

Soil type	Viscosity index (I_{va})
sandy Silt	0.018
Silt	0.025-0.032
clayey Silt	0.015-0.038
silty Clay	0.017-0.034
Clay, Medium plasticity	0.03
Clay, High plasticity	0.04
Clay (bentonite)	0.06
Peat	0.07

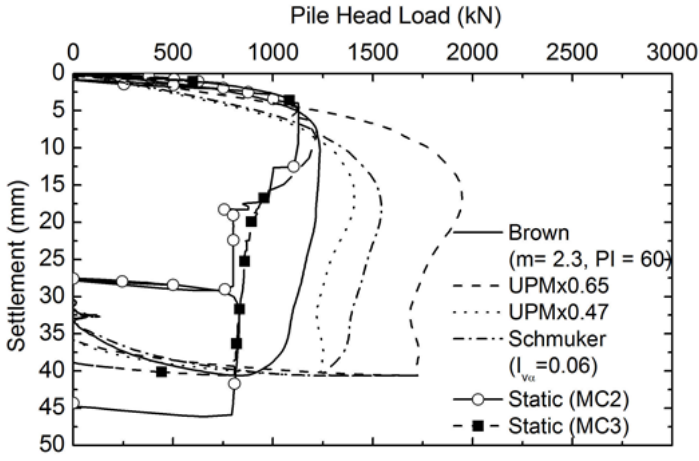


Fig. 6 Comparison of RLT analysis techniques with measured static pile resistance for a cast insitu CFA pile (R1), (Brown and Powell 2012^a)

For the CFA pile the approach proposed by Brown appears to give the best prediction of peak static capacity. The other approaches do not perform as well although the value of UPM correction factor of 0.47 seems more appropriate in this case. Optimisation of the rate parameters to suit the very high plasticity soil results in a UPM correction factor of 0.38 which is a greater correction than the values previously proposed. Similarly in the Schmuker method the limited guidance on parameter selection would suggest values in the range 0.04-0.06 but again a larger optimised correction was required at 0.082. This is outside the range of values given in Table 2.

To allow direct comparison between the analysis of the cast insitu CFA pile and the driven displacement pile identical rate effect parameters were used (as the soil is identical in each case). In contrast the results of analysis on the driven pile show significant under prediction of peak equivalent static capacity for both the Brown and Schmuker techniques (Fig. 7). The UPM approach adopting a correction factor of 0.65 performs the best with a 14% over prediction of static capacity. The peak capacity predicted by the Brown Method is only 65% of that measured.

The apparent under prediction of pile capacity by the analysis techniques for driven piles is caused by the techniques being unable to distinguish between different piles types and installation techniques. For example the Brown technique (Eq. (4)) was developed based upon auger bored and CFA cast insitu piles supplemented by high speed laboratory model pile/ probing tests, hence the good agreement with the CFA piles in this study. The Schmuker method has its origins in low strain rate laboratory element testing (Krieg and Goldscheider 1998). It would also appear that the majority of the piles used to develop the 0.65 factor found were displacement

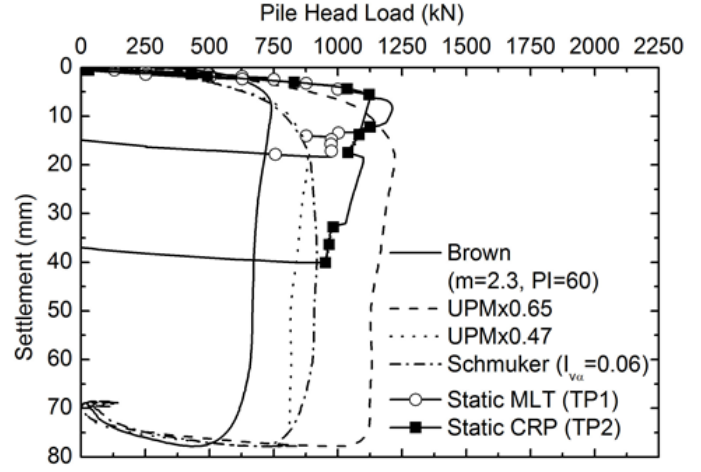


Fig. 7 Comparison of RLT analysis techniques with measured static pile resistance for a driven precast pile (S2), (Brown and Powell 2012^a)

piles and would thus explain the better performance of UPM for the driven piles in this study whilst adopting a factor of 0.65. The more recently proposed UPM correction of 0.47 (Weaver and Rollins 2010) performs better for the CFA piles rather than the driven which is again due to the correction being developed for cast insitu piles only.

In clays it is well known that driving piles may enhance the shaft capacity typically by 30%, with cast insitu techniques only displaying 70% of the shaft capacity obtained from a driven pile (Fleming et al. 2009). This effect was highlighted earlier in the paper by the variation in total stress adhesion factors (Fig. 2). For example reducing the measured static peak capacity (Stage B) of the driven pile TP2 (Fig. 7) to 70% of its measured static capacity to 795 kN brings the results well within the limits of the static prediction (from RLT analysis). Thus the difference between the predicted equivalent static capacity of the driven pile and that measured (Fig. 7) may be assumed to be explained by the difference between the static capacities typically encountered when comparing cast insitu non-displacement piles to driven piles and not as a result of a variation in rate effects associated with differences in pile installation techniques. As noted earlier viscous rate effect parameters were found to be unaffected by pile installation technique when analysing the results of high rate CRP tests (CRP(H)). Thus, assuming that non-displacement piles only display 70% of the driven equivalent static capacity leads to the modification of Eq. (3) for the assessment of the ultimate capacity of driven piles:

$$\tau_{lim} = 1.3\tau_s \left[1 + m \left(\frac{\Delta v}{v_0} \right)^n \right] \quad (7)$$

$$\frac{\tau_{lim}}{\tau_s} = 1.3 + 1.3m \left(\frac{\Delta v}{v_0} \right)^n \quad (8)$$

Such an approach allows the original database of viscous parameters to be utilised for analysis. It is acknowledged that increasing the static shaft capacity in the analysis by 30% to reflect the enhancement due to driving is a simplistic approach. It is also acknowledged that assessing the effects driving has on pile capacity is relatively complex and difficult to predict accurately with complex analysis techniques still relying heavily on empirical correlation (Randolph 2003).

The results of applying Eq. (4) modified to incorporate the “30% enhancement” in the form shown in Eqs. (7) and (8) are shown in Fig. 8. What the approach appears to suggest is that the magnitude of the rate effect is relatively unaffected by the driving process and it is only the enhancement of the static pile capacity due to driving that is causing the differences in the results shown in figures 7 & 8. This observation is tentative as slight variations in the rate effect will be masked by the accuracy of the “30% enhancement”. Optimisation of the results suggests that the enhancement of capacity due to the pile being driven is greater than 30% and is actually better represented by a 35% enhancement. To highlight the improvement to the Brown technique the UPM and Schmucker methods are shown with correction factors optimised to suit the very high plasticity clay based upon the CFA testing results (Fig. 6) but not the “30% enhancement” (Fig. 8).

As previously mentioned the UPM correction factor of 0.65 works well for the driven piles (Fig. 8.) with optimisation in the high plasticity London Clay giving a value closer to 0.62. Reduction of this optimised value to 65% of its original magnitude (i.e. assuming 35% increase in static pile capacity for driven piles) suggests a correction factor μ for a cast insitu pile of 0.40 which is close to 0.38 derived for cast insitu testing in the very high plasticity clay (Brown and Powell 2012). Again this highlights that

a) The UPM analysis must take into account the method of pile installation but that by adjusting the existing parameters it may be possible to simply estimate a correction factor appropriate for various pile installation techniques.

Similarly the viscosity index proposed by Schmucker reduces from 0.082 to 0.054 to suit the analysis for driven piles. This new viscosity index value for the driven pile is closer to values recommended for high plasticity clay (0.04) and organic clays and bentonite (0.06) (Krieg and Goldscheider 1998) which are assumed to be similar to the very high plasticity soils encountered at this site. The Schmucker viscosity index values have previously been criticised for being too low when selected based upon soil type (Brown and Powell 2012). This has been attributed to the relatively low velocities used in the laboratory tests when deriving the parameters. The reduced viscosity index value of 0.054 obtained for driven piles above appears to fit with the parameters proposed by Schmucker but this is thought to be purely coincidental based upon the origins of the method.

Thus rather than suggesting that the published parameters for the various RLT analysis techniques are appropriate for all pile types it seems more appropriate to use them for the specific pile types and installation methods that they have their origins in. For example when testing in fine grained soils current UPM and Schmucker correction parameters are more appropriate for driven or displacement piles and those proposed for the Brown method seem to work for cast insitu or non-displacement piles. Therefore further investigation in to the analysis of RLT tests in fine grained soils must distinguish between different pile and installation techniques and be based upon case study information or testing that accurately models pile installation.

5 CONCLUSIONS

Based upon this study it would seem appropriate that the analysis of RLT must acknowledge the type of pile installation which is being tested. For the RLT and static CRP tests shown it would seem that there is no discernible difference between the rate effects experienced in the RLT testing of driven precast piles and cast insitu piles. The differences in RLT analysis performance observed seem to be as a result of the enhanced static pile capacity often associated with the installation of driven piles in clays. As current analysis techniques in the majority are based upon empirical correlation with static pile tests it is important that future developments and application of RLT analysis acknowledge the potential difference in static capacity that may occur

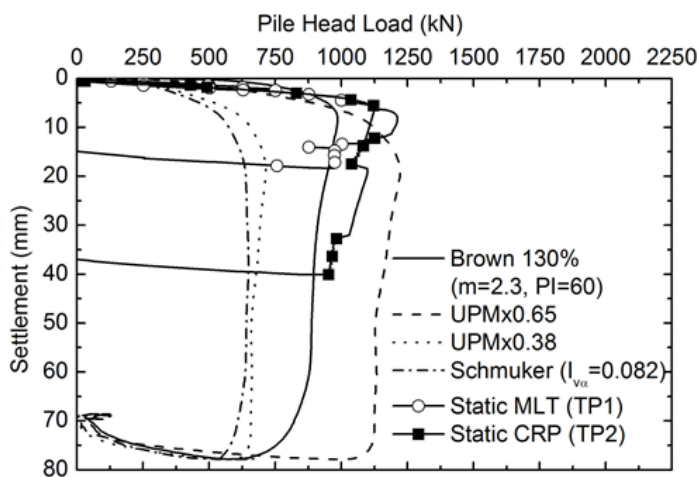


Fig. 8, Comparison of RLT analysis techniques modified to suit driven pile installation with measured static pile resistance for a driven precast pile (S2), (Brown and Powell 2012^a)

for different pile installation methods in different soils.

Existing UPM correction parameters for clays appear to have their basis predominantly in the testing of driven piles and should be applied to other pile types with caution. Ideally new correction factors should be derived that are appropriate to a particular pile installation technique. In the absence of this it may be appropriate to increase the effect of the UPM correction factor to reflect the reduced static capacity associated with cast insitu piles. A similar approach may also be used to modify the analysis proposed by Brown & Hyde (2008) which would allow the use of existing soil specific rate parameters. In both cases this requires the ability to derive the difference between driven and cast insitu static pile capacity prior to testing which is far from straightforward. The Schmucker method also appears to require further development to derive appropriate rate correction factors that are suitable for RLT.

At the current level of understanding of RLT analysis it would seem appropriate to recommend that where RLT is specified there should be documented experience of testing and analysis in both that soil type and for the pile type and installation method proposed. This recommendation seems appropriate until there is greater documented experience of RLT use for a wide range of soil and pile/installation types.

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